Technical Memo

WEST Consultants, Inc.
2601 25th St. SE
Suite 450
Salem, OR 97302-1286
(503) 485 5490
(503) 485-5491 Fax
www.westconsultants.com

DRAFT

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To: Tom Gower, P.E., City Engineer

Company: Gibbs & Olson, Inc.

Date: November 1, 2016

Cc: David Vorse, Public Works Director
City of Castle Rock, WA

From: Hans R. Hadley, P.E., CFM
Senior Hydraulic Engineer

Subject: Al Helenberg Boat Launch Velocity Reduction Structure Alternatives Analysis

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Introduction

The Al Helenberg Memorial Boat Launch was constructed in 2010 approximately 1,300 feet upstream of the State Route 411 Bridge to provide access to the Cowlitz River. A project location map is shown in **Figure 1** (all figures are provided in **Appendix A**). Boaters indicate that during larger wintertime river discharges, high streamflow velocities at the boat launch make use of the upstream lane of the ramp difficult or even dangerous. In an effort to address these concerns, the City of Castle Rock would like to implement a project that will reduce streamflow velocities at the ramp to improve both safety and access for boaters. At the same time, the City would like the sedimentation conditions at the ramp to not be made worse by the project (if possible). In support of this effort, WEST Consultants, Inc. (WEST) was contracted by the City Engineer, Gibbs & Olson, to perform hydraulic analyses of multiple alternative velocity reduction structures. The purpose of the evaluation is to understand the relative ability of each alternative to reduce streamflow velocities at the launch site and its potential effect on sedimentation conditions. The final selected alternative consists of three reinforced 30-ft long concrete panels supported by a combination of vertical and battered steel H-piles. The locations of the recommended panels are shown in **Figure 2**.

Site Reconnaissance

A site reconnaissance was conducted on August 30, 2016. Observations of the channel and floodplain area were made and documented with color photographs (**Appendix B**). The

Manning's 'n' roughness value for the channel is estimated to be 0.03. The Manning's n roughness value for the left (south) overbank is estimated to be 0.12. The Manning's n roughness values for the right (north) overbank are estimated to range between 0.04 and 0.12. Overbank roughness values were estimated based on the investigator's judgment and experience. It is recognized that the Cowlitz River transports a significantly large amount of easily transportable sand size material during high flow events. It is also recognized that changes in bedform morphology can occur with variations in flow. As flows increase the bedforms change from dunes to plane bed, resulting in lower 'n' values during significantly larger flows. Therefore, the channel roughness value was assumed to be slightly larger than the 0.025 value used by the Corps of Engineers' in their high flow HEC-RAS model.

Riprap with a median diameter of about 18-inches was observed along the toe of the banks upstream and downstream of the ramp. The extents of the riprap is unknown as there is significant sediment and vegetative cover. Riprap was also observed along the upstream face of the ramp. It is understood that this material was placed in this location to replace the material that had eroded during the first winter following the completion of the boat launch. Riprap was also observed along the left bank of the channel. It is understood that this was placed to provide erosion protection for the levee.

Sediment deposits were observed beneath the boarding floats. The median bed material size was observed to be coarse sand (D_{50} = 1 mm). Sediment deposits were also observed along the banks adjacent to the ramp. However, the majority of this material appeared to have been recently side-cast as part of the ramp cleanup effort following the December 9, 2015 high water. Photos from both during and after the December 9, 2015 flood were provided by the City and are shown in **Appendix B** (Photos 13-16).

Survey

Bathymetric survey of the channel was conducted in August and September of 2016 by Gibbs & Olson. High density survey capable of supporting the development of a 2-dimensional model was collected from approximately 2,150 feet upstream to 900 feet downstream of the ramp. Four channel cross sections were also surveyed in the 900-foot reach immediately downstream of the high-density survey in order to provide additional data needed for the development of the 1-dimensional hydraulic model. The horizontal coordinate system for the survey is NAD 83 Washington State Plane South Zone, US Foot. The vertical datum for the survey is the North American Vertical Datum of 1988 (NAVD 88).

Hydrology

USGS Gage *142430000 Cowlitz River at Castle Rock, WA* is located approximately 1,400 feet downstream of the ramp at the Hwy 411 Bridge (A Street) and has a period of record of 90 years (1926 to present). Mean daily flow records are available for the prior 10 years (2006 – 2016) and mean stage are available for the prior years. Mean daily flow data for the 2006 – 2016 period were plotted and reviewed, and a flow duration curve developed (**Figure 3**). Based on review of the flow data, three flows were chosen to be simulated in the hydraulic models:

- 30,000 cfs – represents the approximate upper limit of usability of the ramp
- 9,000 cfs – represents a typical winter flow rate
- 5,000 cfs – represents a typical summer flow rate

Sediment Transport

Sediment transport conditions in the Cowlitz River are highly influenced by the delivery of sediment from the Toutle River, which flows into the Cowlitz River about 2.4 miles upstream of the project site. The Toutle River continues to deliver significant quantities of silt- and sand-sized sediment as a result of continued erosion of the debris avalanche created by the eruption of Mount Saint Helens in May 1980. The U.S. Army Corps of Engineers has continued to manage sediment in the Toutle River, most recently raising the spillway elevation at the Sediment Retention Structure to improve the sediment trapping efficiency. They have also dredged the lower portion of the Cowlitz River near the confluence with the Columbia River. However, significant quantities of sand-sized material will continue to be transported through the project reach as both bed load and suspended load.

Much of the fine sediment in the Cowlitz River is transported as suspended load during high flow events. As a result, sediment deposition occurs in areas of low velocity and low shear stress such as the areas along the banks, the inside of channel bends, and near obstructions to flow. During the December 2015 high water event (Q = 83,700 cfs), significant sedimentation occurred at the boat launch. The surface of the ramp was buried in as much as 2.5 feet of sand-sized material. Post-flood photographs indicate that some amount of the deposited sediment located near the base of the ramp was eroded as the river's discharge decreased. However, a significant amount of sediment remained in the immediate vicinity of the transverse floats, causing them to be partially grounded during low water conditions.

A significant portion of the sediment that was deposited at the ramp was likely conveyed by the river as suspended load. Therefore, significant changes in velocity and shear stress at and near the ramp location should be expected to affect the sedimentation conditions. The primary objective of this project is to increase boater safety by reducing velocities during high water conditions. Alternatives that result in significant velocity reductions and/or create an eddy would be expected to increase the rate of sediment deposition.

1-Dimensional Hydraulic Modeling

The primary purpose of the 1-dimensional model is to provide a starting downstream boundary condition for the 2-dimensional hydraulic model. This removes the expense of collecting additional high density survey data needed to extend the 2-dimensional model downstream to the Hwy 411 bridge. The 1-dimensional model can also be used in the future for developing a FEMA Conditional Letter of Map Revision (CLOMR) and Letter of Map Revision (LOMR) that would be required should the proposed project move forward to final design and construction.

HEC-RAS version 5.0.1 software (USACE, 2016) was used to develop an existing conditions steady state hydraulic model for the Cowlitz River in the vicinity of the project site. The upstream boundary of the model is located approximately 2,150 feet upstream of the ramp. The downstream boundary of the model is located approximately 1,350 feet downstream of the ramp and is coincident with USGS gage 142430000 at the downstream face of the highway 411 Bridge. As seen in Figure 1, a total of 15 cross sections are used in HEC-RAS to represent the geometry of the channel and floodplains. The cross-section geometry is based on the Gibbs & Olson survey within the channel and LiDAR data from the U.S Army Corps of Engineers for the overbanks. The downstream boundaries for the three simulated flows are specified as known water surface elevations based on rating curve data from the USGS gage.

Two-Dimensional Modeling

Since the purpose of the project is to modify flow velocities, evaluation and development of the various conceptual designs requires detailed information about the effects of the structures on local flow dynamics. A 2-dimensional hydraulic model was developed to evaluate potential changes in the magnitude and direction of flows and magnitude of shear stresses in the vicinity of the ramp.

The two-dimensional hydrodynamic software modeling program Sedimentation and River Hydraulics – Two-Dimensional (SRH-2D) Version 3.1.1 (dated July 2016), developed by the US Bureau of Reclamation (USBR), was used to simulate the hydraulic conditions of the Cowlitz River near the project site.

The model mesh was developed using the Surface-water Modeling System (SMS) Version 11.2.9 (SMS) developed by Aquaveo (2015). Model development involved the following steps:

1. Development of a conceptual model using arcs (polylines) to parse the modeled area into multiple zones defined by unique characteristics such as land use, Manning's n hydraulic roughness value, and specific project sites.
2. Assignment of mesh node spacing for each zone. The mesh node spacing varies significantly within the computational domain depending on the resolution required, with larger spacing in the floodplain and significantly smaller spacing in the channel where more detailed model output is required. Spacing ranges from 5 feet near the project site to 50 feet along the periphery of the floodplain.
3. Interpolation of topographic data points to the mesh. Topographic data in the SRH-2D model are based on the DTM developed for the project area.
4. Assignment of a downstream boundary condition. A water surface elevation boundary condition was assigned in SRH-2D that was equal to the water surface elevation at Cross Section 479 in the one-dimensional HEC-RAS model for each evaluated flow.
5. Pre-processing of model input data (mesh, inflow and outflow parameters, monitor lines, simulation times, output intervals) using the SRH-2D pre-processor to create the input files for the model.

Table 1 shows the Manning's n values for each land use type specified in the SRH-2D model.

Table 1 –Manning’s n Values

Land Use Type	Manning’s n Value
River	0.03
Field/Open	0.04
Pavement	0.015
Dense Residential/Commercial	0.12
Forest	0.1
Rural Residential	0.08

An existing conditions model was initially developed to provide a basis for comparison of the potential alternatives. It should be noted that the existing piles and floating logs used to deflect debris away from the ramp are not represented in the model. Based on the simulated flow depths and velocities, computational limitations of the model prohibit representation of very small features such as individual piles, without introducing model instabilities. As the piles currently represent a very small blockage to flow relative to the entire channel cross section it is assumed that they have very limited impact on flow characteristics and that the existing conditions model is a good representation of existing flow patterns. The existing conditions model was then modified to represent 16 conceptual variations of potential velocity reduction structures. The design variations are all based on the general premise of steel H-piles driven into the stream bed which would be used to support precast reinforced concrete panels between the H-piles. The panels would be keyed into the channel bed and would have top elevations of 43.0 ft, which is approximately 1 foot above the 30,000 cfs flow elevation. The design variations ranged in location from immediately upstream of the ramp to a point approximately 300 feet upstream of the ramp (approximately 200 feet upstream of the existing debris deflector). The designs also considered various alignments, solid vs. discontinuous panels, and various panel and total structure lengths. The use of a single structure vs. multiple structures in tandem was also evaluated.

Based on review of the model output from the 16 modeled alternatives, Concept 7d is considered the preferred alternative. Concept 7d, located approximately 300 feet upstream of the ramp, is approximately 150 feet long, angled approximately 45 degrees to the channel bank in a downstream direction, and consists of three 30-foot long panels with 30-foot spacing between the panels (**Figure 2**).

Results

Simulated velocities for the 30,000 cfs, 9,000 cfs, and 5,000 cfs flows under existing conditions are provided in **Figure 4**, **Figure 5**, and **Figure 6**, respectively. Simulated shear stresses for the 30,000 cfs flow under existing conditions are shown in **Figure 7**. Simulated velocities for the 30,000 cfs, 9,000 cfs, and 5,000 cfs flows for the preferred alternative are provided in **Figure 8**, **Figure 9**, and **Figure 10**, respectively. Simulated shear stresses for the 30,000 cfs flow for the preferred alternative are shown in **Figure 11**. The 2-D model results indicate that the preferred alternative would reduce velocities near the end of the boarding floats from 5 ft/s to 3 ft/s for the 30,000 cfs flow. Velocities at the end of the boarding floats would be reduced from 2.3 ft/s to 1.3 ft/s for the 9,000 cfs flow and velocities would remain approximately the same for the 5,000 cfs flow. The reduction in velocities is favorable from a hazard perspective and would provide a safer ingress/egress zone for boaters, particularly during higher flow conditions.

Model results indicate that the proposed structure will not create an eddy which, as previously mentioned, would likely exacerbate existing sedimentation issues. However, it is noted that shear stresses would be reduced downstream of the proposed structure which may result in an increase in sedimentation during high flow events when the Cowlitz River is transporting a significant suspended sediment load. Incipient motion calculations using Shield's equation suggests that the coarse sand (1 mm) is mobilized for shear stress values greater than 0.02 lb/ft^2. The existing conditions model results indicates that shear stress values near the base of the ramp and the transverse floats are about 0.27 lb/ft^2 and 0.32 lb/ft^2, respectively for a discharge of 30,000 cfs. The preferred alternative model results indicate that the shear stress values near the base of the ramp and the transverse floats are about 0.08 lb/ft^2 and 0.18 lb/ft^2, respectively for a discharge of 30,000 cfs. For both existing conditions and the preferred alternative, shear stress values at the upper portion of the ramp are less than 0.01 lb/ft^2. Sedimentation conditions along the upper portion of the ramp are not expected to change significantly as a result of the project. Although the shear stress values will be greater than required to transport coarse sand-sized material, sediment deposition rates for the area near the base of the ramp and the transverse floats are likely to increase for the proposed alternative compared to existing conditions due to the excessive supply of sediment delivered by the Toutle River. Periodic sediment removal will likely be required.

The proposed project is expected to change the flow directions and velocities in the immediate vicinity of the structure. As seen in **Figure 8**, the velocity along the bank is expected to increase compared to the existing conditions. This area (approximately 100 ft long) may require bank protection if the existing bank is not sufficiently protected. Bank protection would likely be in the form of a riprap revetment or combination of riprap toe and vegetation. Additional reconnaissance for this location is recommended to determine the adequacy of the bank material to resist erosion.

The portion of the proposed structure located furthest from the bank will create a zone of lower velocity immediately downstream. This area is likely to accumulate sediment over time. However, the structure was location 300 ft upstream of the ramp to lessen the chances that the sediment deposition in this low velocity zone would extend to the ramp location.

Minor reductions in shear stress occur along the right (west) bank for a distance of about 100 ft upstream of the structure. Additional sediment deposition may occur in this area. Shear stress values for areas further upstream were not significantly changed by the proposed project. Significant changes to the channel morphology or bank erosion potential are not expected to occur for this area.

Increases in velocity and shear stress are expected to occur immediately east of the structure as more of the flow is directed toward the center of the channel. In this location, additional scour of the channel bed is likely to occur.

The proposed project is located within a regulatory FEMA floodplain and floodway. According to FEMA regulations, the project should not cause a rise in the regulatory floodplain and floodway elevations. A complete no-rise hydraulic analysis based on the FEMA Base Flood was not conducted as part of the alternatives analysis detailed in this memo; however, the modeling conducted for the alternatives analysis indicates that that the proposed alternative is likely to increase water surface elevations for the Base Flood. A no-rise analysis using FEMA methodology will need to be conducted for the chosen alternative. If a rise is shown to occur, the project will require a Conditional Letter of Map Amendment (CLOMR) to be submitted by the City to FEMA prior to implementation of the project. Following completion of the project, a Letter of Map Revision (LOMR) will need to be submitted by the City to FEMA.

If you have any questions, please do not hesitate to contact me at 503-485-5490.

APPENDIX A
FIGURES

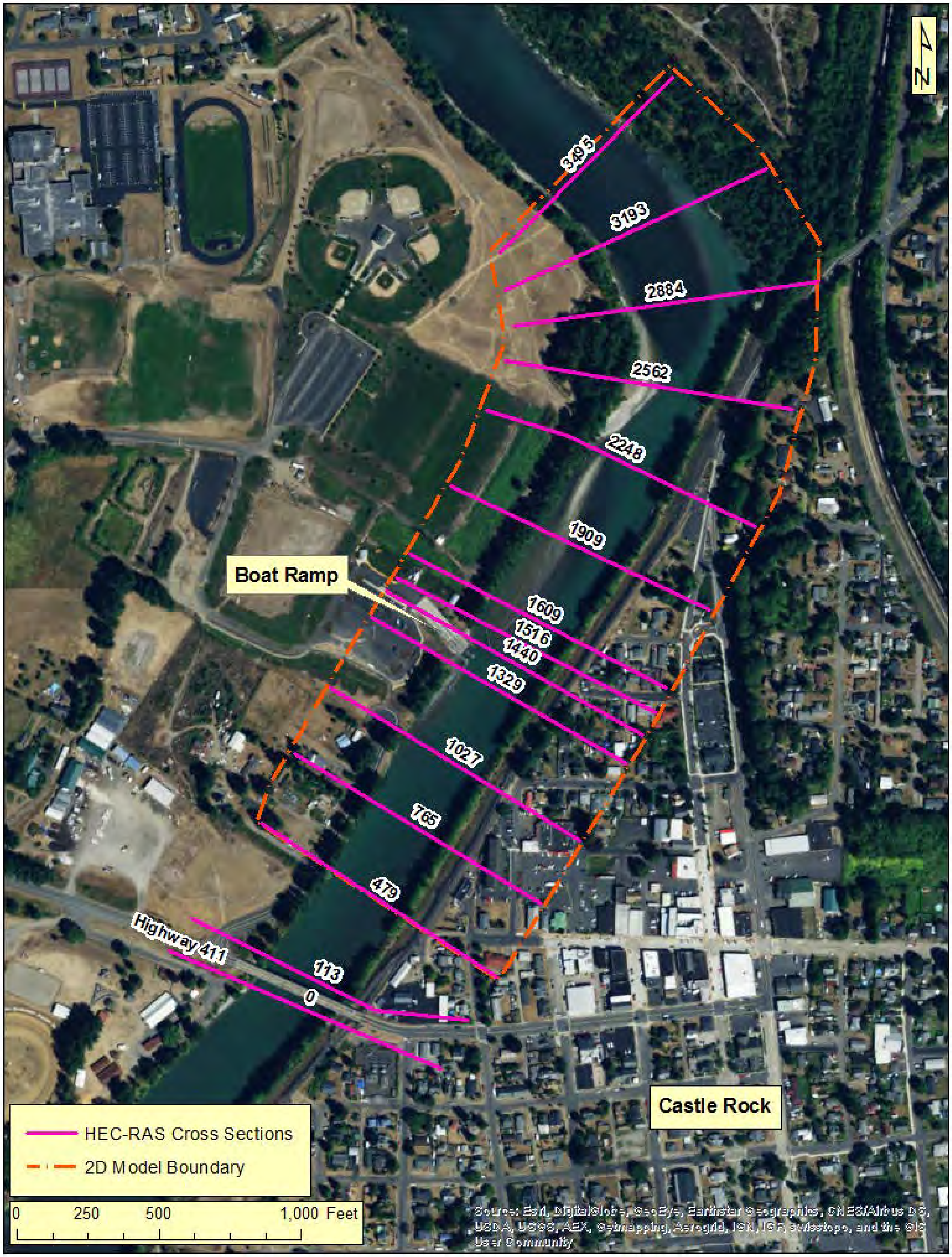

Figure 1. Location Map

Figure 2. Preferred Alternative (7d)

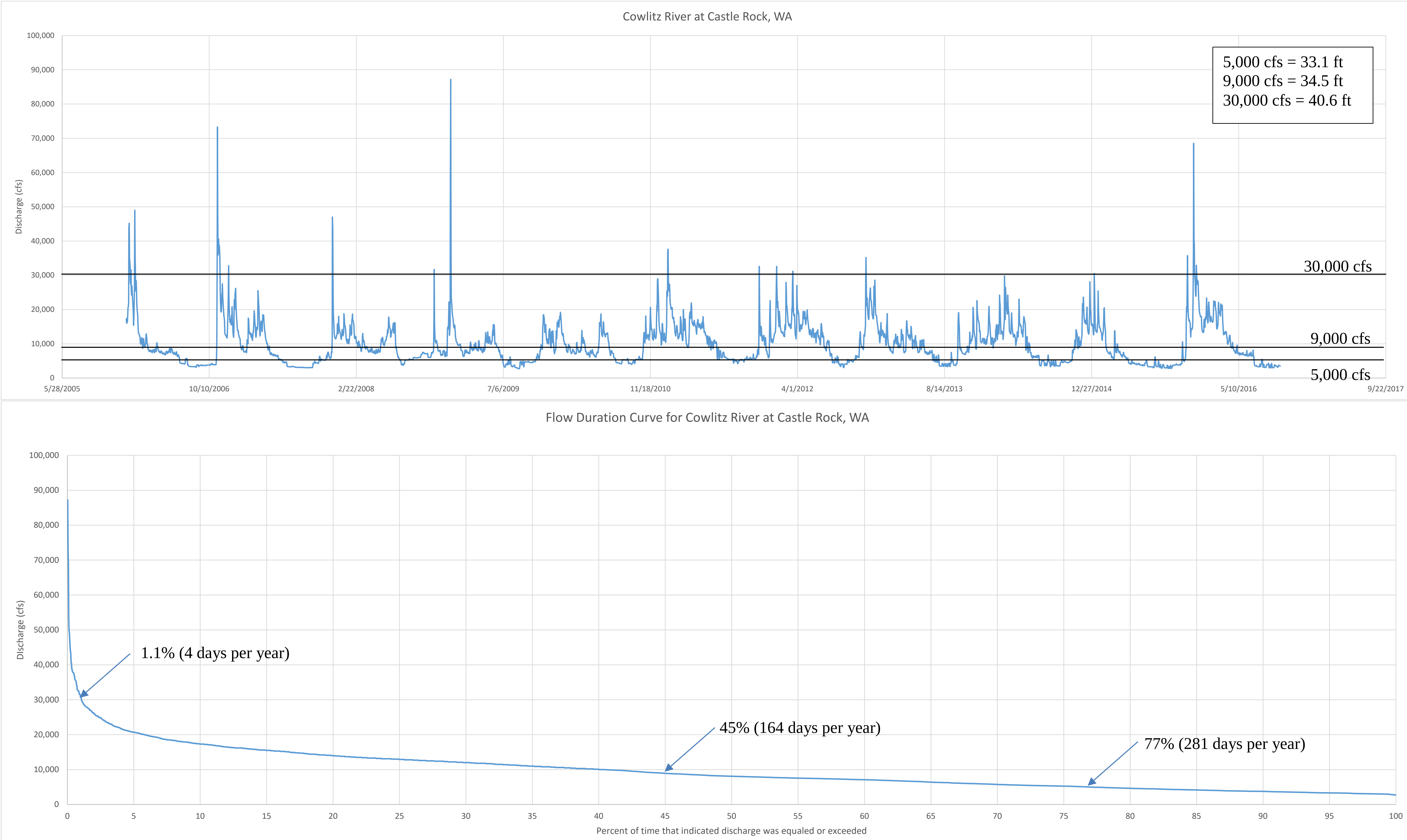

Figure 3. Mean daily flows from 2006-2016 and associated flow duration curve

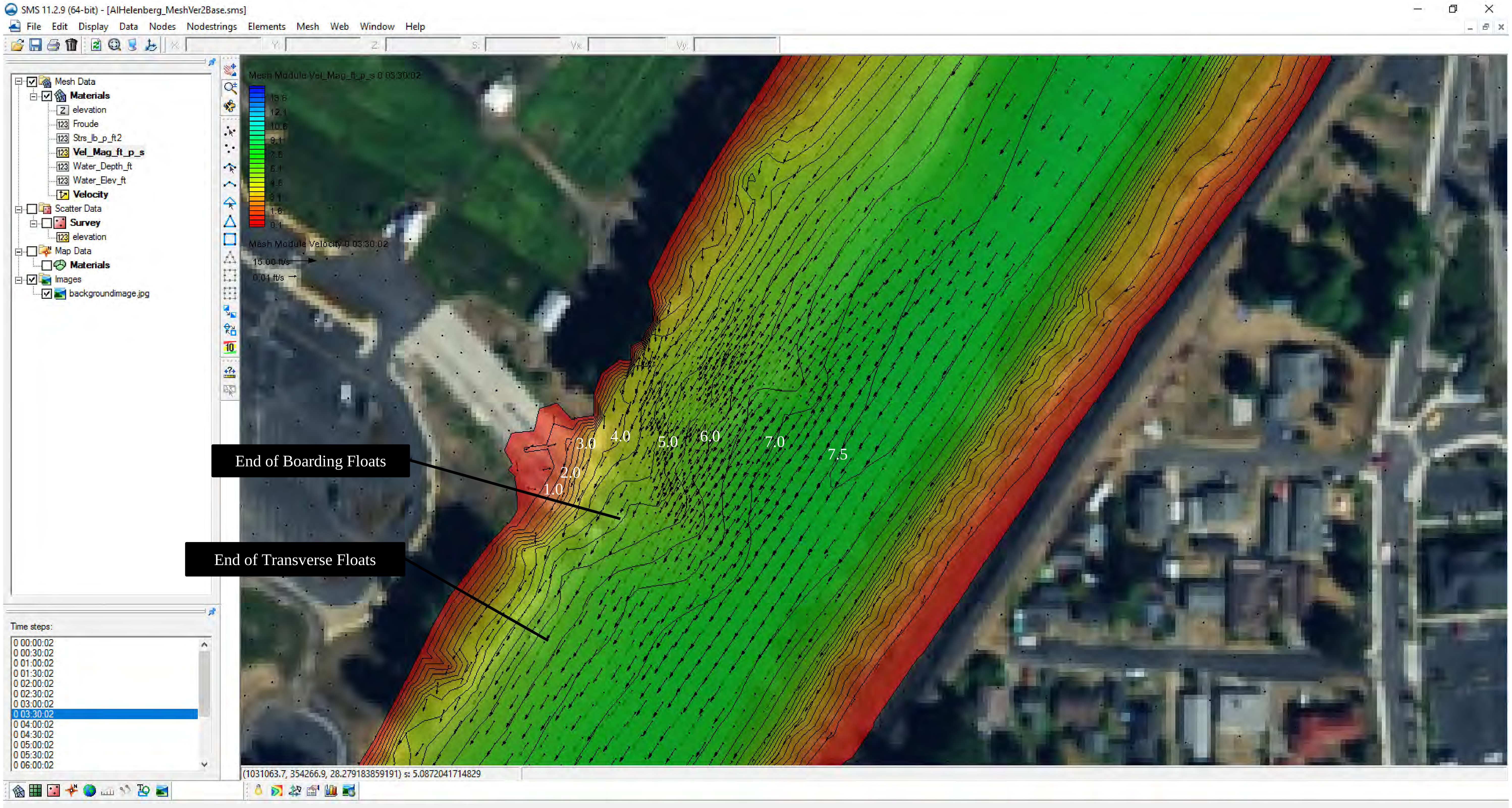

Figure 4. Existing conditions velocities (30,000 cfs)

Figure 5. Existing conditions velocities (9,000 cfs)

Figure 6. Existing conditions velocities (5,000 cfs)

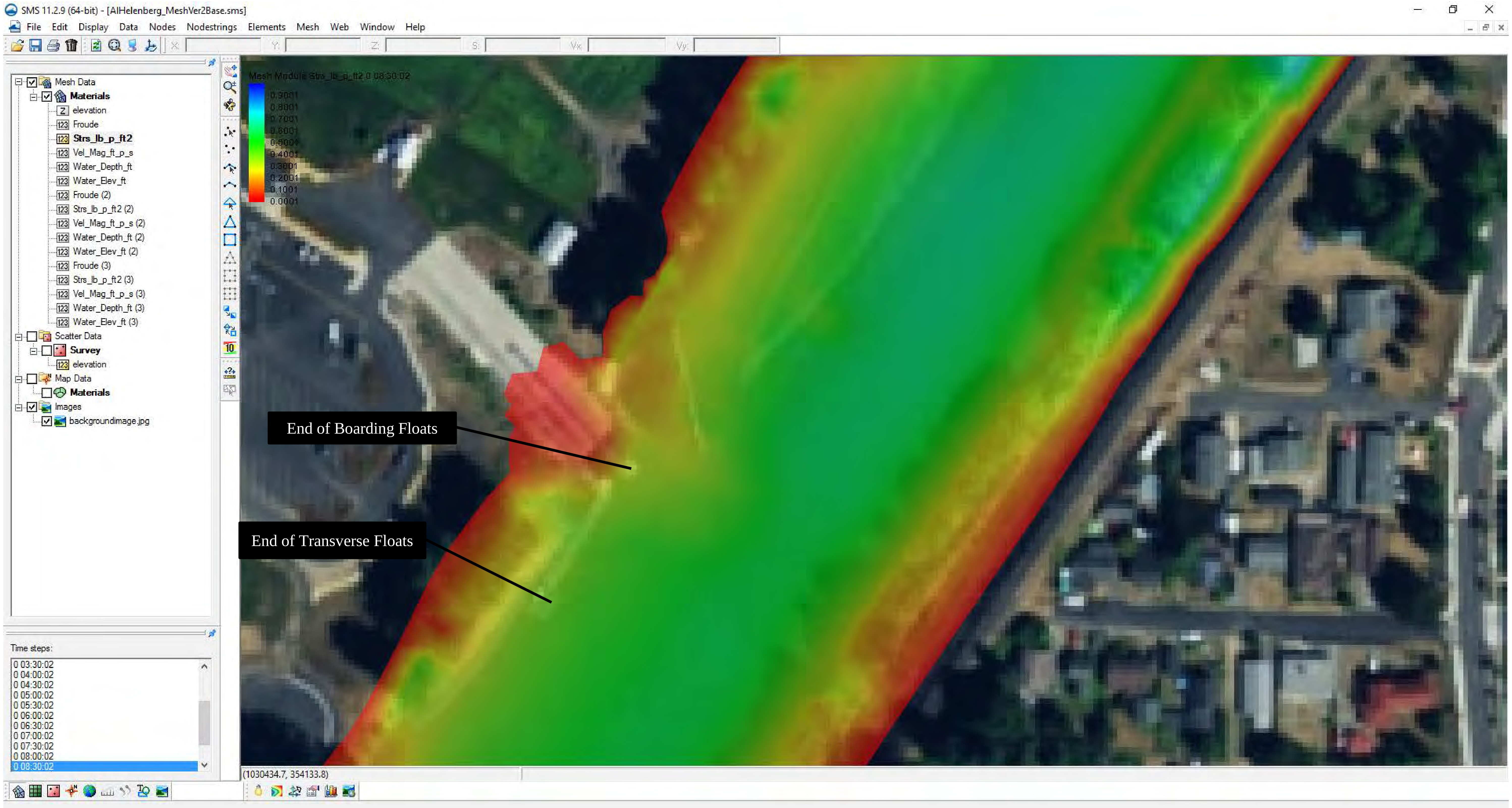

Figure 7. Existing conditions shear stresses (30,000 cfs)

Figure 8. Preferred Alternative (7d) velocities (30,000 cfs)

Figure 9. Preferred Alternative (7d) velocities (9,000 cfs)

Figure 10. Preferred Alternative (7d) velocities (5,000 cfs)

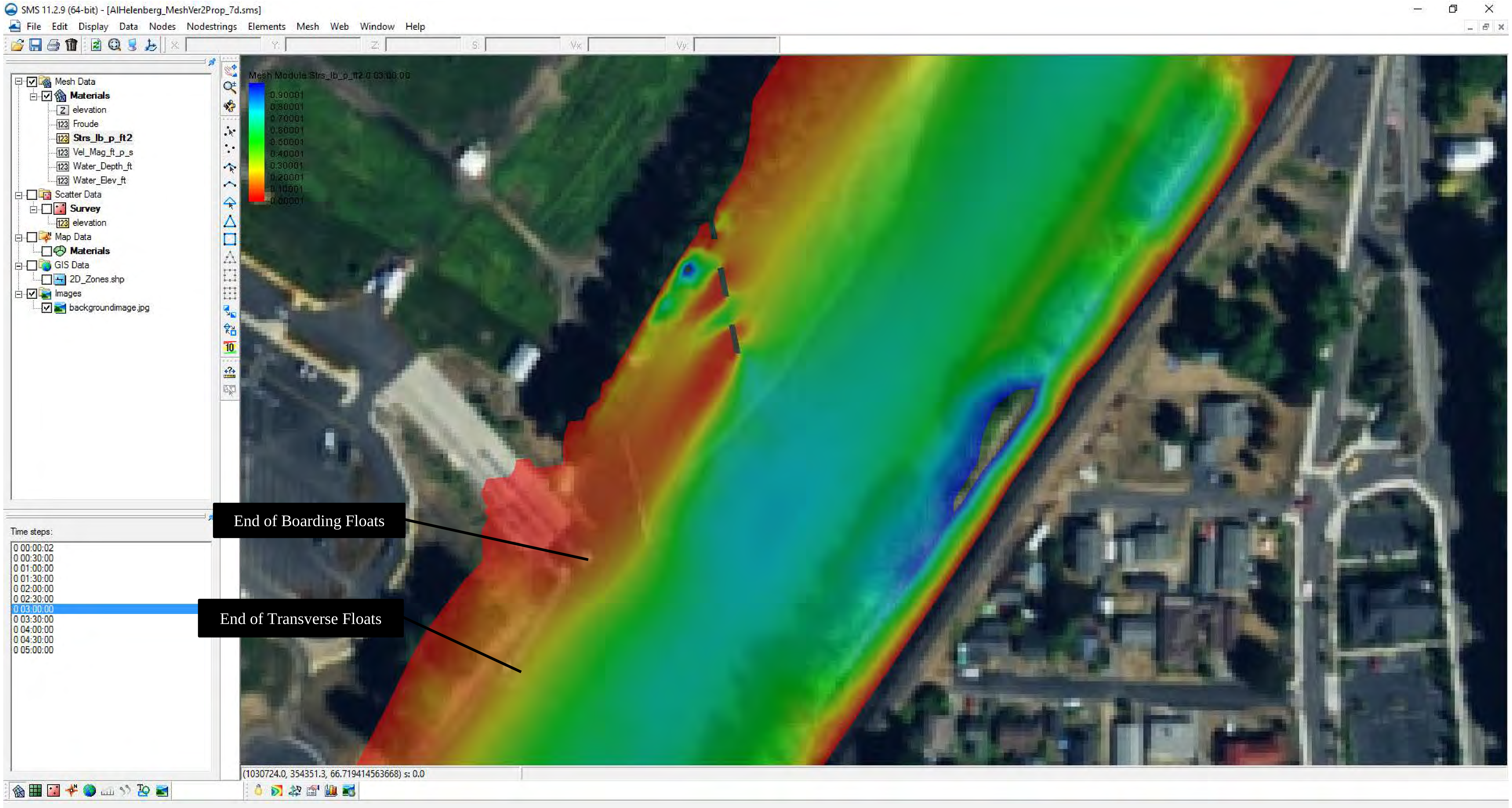

Figure 11. Preferred Alternative (7d) shear stresses (30,000 cfs)

APPENDIX B
PHOTOGRAPHIC LOG

Photo 1 – View of boat ramp from parking area

Photo 2 – View from top of ramp

Photo 3 – Riprap protection added after Dec 2015 high water

Photo 4 – Looking upstream along left bank

Photo 5 – Pile and log debris deflector

Photo 6 – Boarding floats (foreground) and transverse float (background)

Photo 7 – Boarding floats and marks from sediment deposition

Photo 8 – Looking upstream from end of boarding floats

Photo 9 – Looking downstream along transverse floats

Photo 10 – looking at base of ramp and debris from Dec 2015 high water

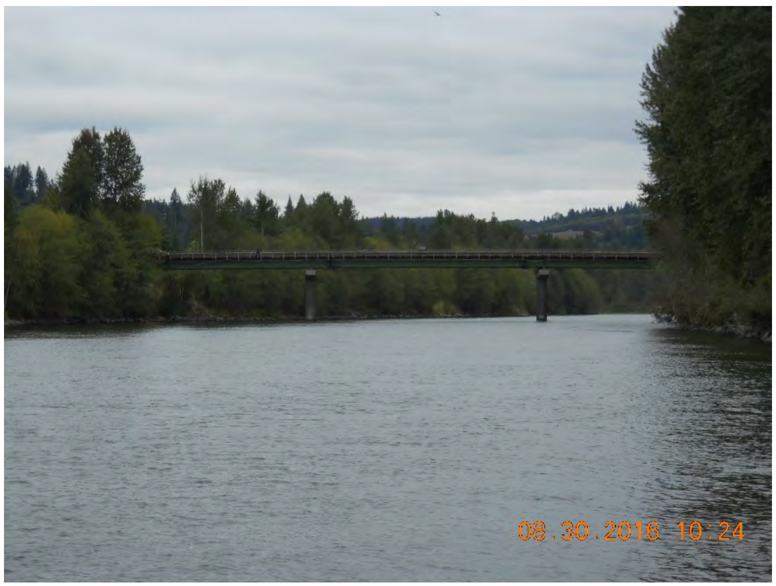

Photo 11 - Looking downstream at Hwy 411 bridge

Photo 12 – riprap protection along downstream bank

Photo 13 – Sediment deposits from December 9, 2015 high water

Photo 14 – Sediment deposits from December 9, 2015 high water

Photo 15 – December 9, 2015 high water

Photo 16 – Debris on floats during December 9, 2015 high water